



***Peto MacCallum Ltd.***  
C O N S U L T I N G   E N G I N E E R S

**FOUNDATION INVESTIGATION AND DESIGN REPORT**

**for**

**LOVERING LAKE ROAD OVERPASS SOUTHBOUND  
HIGHWAY 69**

**SITE NO. 46-508/2, W.P. 5261-05-01**

**DISTRICT 54, SUDBURY, ONTARIO**

***PHASE 1, STA. 12+200 TO 15+400  
TOWNSHIP OF SERVOS***

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PML Ref.: 06TF052B  
Index No.: 411FIR and 412FDR  
GEOCRES No.: 41I-213  
November 13, 2007



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**FOUNDATION INVESTIGATION REPORT**

for  
Lovering Lake Road Overpass Southbound  
Highway 69  
Site No. 46-508/2, W.P. 5261-05-01  
District 54, Sudbury, Ontario

*Phase 1, Sta. 12+200 to 15+400  
Township of Servos*

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**1. INTRODUCTION**

This report summarizes the results of the foundation investigation carried out for the proposed new Lovering Lake Road Overpass Southbound on the realigned Highway 69 about 44 km south of Sudbury. The investigation was conducted for Totten Sims Hubicki Associates Limited (TSH) on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed Lovering Lake Road overpass carries the new Highway 69 southbound lanes (SBL) between approximate Sta. 14+144.3 and 14+171.0.

This report provides subsurface information pertaining to the proposed Lovering Lake Road SBL overpass and approaches within about 20 m of the abutments, between approximate Sta. 14+124.3 and 14+191.0.

**2. SITE DESCRIPTION AND GEOLOGY**

The site is located approximately 44 km south of Sudbury at Lovering Lake Road. The existing Lovering Lake Road is a winding gravel road leading east from the existing Highway 69 to local cottages. Land use also includes forestry exploration.

The terrain has a general rugged topography with numerous massive rock outcrops. The local topography is irregular and comprises areas of wooded area ground separated by steep rock ridges. Soil cover over the rock outcrops is generally sparse. Scattered boulders are also present at the ground surface of the site in particular at the south limit where a talus slope is noted at the base of a prominent rock outcrop.



The overpass site area is generally located in the Precambrian Laurentian peneplane. In general, Metasedimentary rocks of the Huronian Supergroup and gneisses of the Grenville Province underlie the alignment. The area has undergone considerable folding, intrusive activity, regional metamorphism and faulting. Numerous bedrock outcrops are visible at the overpass site location.

### **3. INVESTIGATION PROCEDURES**

The field work for the north and south abutments for SBL overpass was carried out during the periods of March 25 to 27, 2007 and April 17 to 19, 2007.

The scope of the subsurface investigation comprised 12 boreholes that were advanced/drilled to depths of 0.2 to 4.1 m at the locations shown on Drawing LLS-1, appended. In addition, four of the boreholes, LLS-4A and LLS-6 to LLS-8, were cored 3.0 to 4.3 m into the bedrock to depths between 4.3 to 6.4 m.

Sutcliffe Rody Quesnel (SRQ) Inc. staked the alignment of the proposed four-lane Highway 69 at the structure location. Peto MacCallum Ltd. (PML) selected the positions of the boreholes along the staked alignment and determined the ground surface elevations at the borehole locations. SRQ Inc. provided the following temporary benchmarks (TBM) established on existing ground level at the working points (WP) for each of the foundation units:

<b>TBM</b>	<b>DESCRIPTION</b>	<b>ELEVATION (*)</b>
TBM1	Existing ground at South abutment W.P.1	209.3
TBM2	Existing ground at North abutment W.P. 2	204.7

(\*) Geodetic, metric

The boreholes were advanced by various methods, as required by accessibility and prevalent weather limitations. Borehole LLS-1 was advanced using a portable power auger, boreholes LLS-2, LLS-3, LLS-4B and LLS-4C were advanced using manual augering methods and boreholes LLS-4 and LLS-4A were advanced using a portable drill (Hilti) equipped for rotary diamond core drilling and washboring due to site access restrictions on the steep slope. The remaining five boreholes, LLS-5 to LLS-9 were advanced using continuous flight solid stem augers powered by a track-mounted CME-55. All of the power equipment were supplied and



operated by specialist drilling contractors. The drilling crews worked under the full-time supervision of a member of our engineering staff.

Representative samples of the soils were recovered at frequent depth intervals. In the boreholes advanced with conventional drill rigs, the samples were obtained using a split spoon sampler in conjunction with standard penetration tests. Where standard penetration tests were not carried out the consistency/relative density of the encountered soils was estimated from manual examination or the rate (ease) of advance of the auger. One penetrometer test was performed on sample 1 of borehole LLS-9. The results of penetrometer tests are typically lower than the actual values due to potential sample disturbance.

Four boreholes, LLS-4A and LLS-6 to LLS-8, were extended 3.0 to 4.3 m into the bedrock using NQ diamond rock coring equipment. Photographs of the rock cores are shown in Appendix A. The boreholes were backfilled in accordance with the MTO guidelines and MOE regulation 903 for borehole abandonment procedures using a bentonite/cement mixture grout.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination, soil classification and laboratory testing. The laboratory test program comprised the following tests:

- Atterberg limits determinations (2)
- Grain size analyses (3)
- Natural moisture content analyses (8)

A third Atterberg limits determination was not carried out due to the insufficient size of the recovered samples. The results of grain size determinations and Atterberg limits are shown on the Record of Borehole sheets. Grain size distribution charts are presented on Figure GS-LLS-1 and plasticity charts are presented on Figure PC-LLS-1.



#### **4. SUMMARIZED SUBSURFACE CONDITIONS**

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations and groundwater observations. Site photographs are provided in Appendix B.

The borehole locations and stratigraphic cross-sections prepared from the borehole data are presented on Drawing LL-S-1.

The depth of the soil cover revealed in the boreholes varies from 0.2 to 4.1 m. The soil cover generally comprises topsoil locally mixed with cobbles and boulders over discontinuous cobbles and boulders (talus slope at south limit) and clayey silt units mantling bedrock. Scattered layers of cobbles and boulders are encountered in the boreholes.

##### **4.1 Topsoil**

Surficial deposits of topsoil locally mixed with cobbles and boulders are present in all boreholes except for borehole LLS-9. The dark brown topsoil layer is 200 to 600 mm thick. Boreholes LLS-2 and LLS-3 terminated by refusal on probable bedrock at 0.2 and 0.6 m depths, elevations 210.0 and 216.3. Borehole LLS-4B and LLS-4C terminated on probable boulders present within a talus slope noted at the base of a prominent rock outcrop. These boreholes terminated at 0.3 m depth, elevations 208.1 and 208.2.

##### **4.2 Sand and Gravel**

A surficial layer of grey sand and gravel mixed with cobbles was encountered in borehole LLS-9 and extend to 0.5 m depth, elevation 204.4.

##### **4.3 Clayey Silt**

Layers of cohesive clayey silt occur below topsoil in boreholes LLS-5 to LLS-8, and below the sand and gravel layer in borehole LLS-9 and extend 1.1 to 2.4 m depths, elevations 202.5 to 204.0. The consistency of the clayey silt material ranges from stiff to very stiff. Penetrometer tests indicated undrained shear strength value of 87 kPa. N values ranged from 10 to 18.



The grain size distribution charts of three samples of the clayey silt material are presented on Figure GS-LLS-1. Plasticity charts of the two samples are presented in Figure PC-LLS-1. The liquid limits were 21 and 28 and the plastic limits 14 and 21, both giving a plasticity index value of 7. Natural moisture content determinations ranged from 17 to 32%.

#### **4.4 Sandy silt**

Localized deposits of cohesionless sandy silt mixed with occasional cobbles and boulders were encountered below topsoil in borehole LLS-1 extending to 1.5 m depth, elevation 222.3 and below a clayey silt layer in borehole LLS-7 extending 1.3 to 1.7 m depths, elevations 203.4 to 203.0. The relative density of the sandy silt soil is inferred to be loose to compact. One standard penetration N value of 50 blows per only 30 mm of penetration was obtained with the sampler bouncing on probable bedrock in borehole LLS-7.

#### **4.5 Cobbles and Boulders**

A localized deposit of cohesionless cobbles and boulders mixed with gravel, sand and silt that is inferred to comprise a talus slope deposit is encountered below the topsoil in boreholes LLS-4 and LLS-4A. The material extends to 2.1 and 2.8 m depths, elevations 206.5 and 207.2. The layer could only be penetrated with portable equipment equipped with rotary diamond coring and washboring in view of the numerous boulders present.

#### **4.6 Bedrock**

A detailed description of the rock cores retrieved from boreholes LLS-4A and LLS-6 to LLS-8 is provided in Table A and summarized on the record of borehole logs. The bedrock comprises light to medium grey to light grey and pink granitic gneiss in the four cored boreholes, LLS-4A and LLS-6 to LLS-8. A localized 0.8 m thick white and black migmatite band was encountered in borehole LLS-7 above granitic gneiss. The rock is typically slightly weathered to unweathered and exhibits medium to high strength on the north abutment and variable very poor to excellent quality under the south abutment site.





At the north abutment (boreholes LLS-6 to LLS-8) the bedrock surface is confirmed by drilling three rock cores 3.0 to 3.2 m long from depths of 1.1 to 1.7 m, elevations 202.8 to 204.0, indicating a maximum surface level difference of 1.2 m between borehole locations. The slope of the bedrock surface between the 3 boreholes is 2 and 20°. Photographs of the rock core taken in boreholes LLS-6 to LLS-8 are shown on Photos 1 to 6, Appendix A.

At the south abutment (boreholes LLS-4, LLS-4A to LLS-4C) the bedrock surface is confirmed by drilling one rock core hole 4.3 m long from depth 2.1 m, elevation 207.2. Rock coring at the originally planned borehole LLS-4 was started at 2.8 m depth, elevation 206.5, however, could not be continued past 4.1 m depth due to a broken coring bit at the bottom of a boulder or on a fracture in the rock. Thus, the borehole was moved one meter south on the centreline of Lovering Lake Road overpass southbound from Sta. 14+144 to 14+143. The bedrock surface was encountered at 2.1 m in borehole LLS-4A and was cored 4.3 m, instead of 3.0 m, to further evaluate the rock condition under the talus slope deposit. Photographs of the rock core taken at borehole LLS-3A are shown on Photos 7 and 8, Appendix A.

In the north and south abutments boreholes, the measured core recovery varies typically between 92 and 100%, with isolated values of 81 and 86% in borehole LLS-4A. The RQD determined from the north abutment rock cores, LLS-6, LLS-7 and LLS-8, is typically greater than 90% (range of 75 to 100%), with two isolated values of 75 and 89% in boreholes LLS-6 and LLS-8, respectively, indicating good to excellent quality rock. The range of RQD for borehole LLS-4A is between 0 and 100%, indicating variable very poor to excellent rock quality.

#### **4.7 Groundwater**

Groundwater was only observed in boreholes LLS-7 and LLS-8 at 0.6 and 0.9 m depth, elevations 203.5 and 204.1 upon completion of the drilling. No groundwater was observed in the remaining boreholes during and upon completion of drilling.

The groundwater is subject to fluctuations at the site due to seasonal conditions and rainfall patterns.



## 5. CLOSURE

The field work was carried out under the supervision of Mr. N. Lee-Bun, and direction of Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer. Marathon Drilling Co. Ltd. and Aardvark Drilling Inc. supplied the soil and rock drilling equipment. The laboratory testing was carried out in the PML laboratory facilities in Toronto.

The report was prepared by Mr. C. M. P. Nascimento, P.Eng., and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Yours very truly,

Peto MacCallum Ltd.

A handwritten signature in black ink, appearing to read "C. M. P. Nascimento", is positioned above the printed name.

C. M. P. Nascimento, P.Eng.,  
Senior Project Engineer



A handwritten signature in black ink, appearing to read "Brian R. Gray", is positioned above the printed name.

Brian R. Gray, MEng, P.Eng.  
MTO Designated Principal Contact



CN/BRG:nr-mi/lmr



TABLE A  
ROCK CORE DESCRIPTION

CORE RECOVERY				CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)
LLS-4A	3	2.1 – 2.5	100	100	2.1 - 6.4
	4	2.5 – 3.0	95	64	
	5	3.0 – 3.7	86	0	
	6	3.7 – 4.4	81	0	
	7	4.4 – 5.0	100	17	
	8	5.0 – 5.2	100	71	
	9	5.2 – 6.4	94	13	
LLS-6	2	1.1 – 2.4	100	75	1.1 – 4.3
	3	2.4 – 2.9	100	100	
	4	2.9 – 4.3	100	93	
LLS-7	3	1.7 – 2.6	92	92	1.7 – 2.5
	4	2.6 – 4.1	100	100	
	5	4.1 – 4.7	100	100	

GRANITIC GNEISS: Light to medium grey and pink, fine to medium crystalline, slight banding (dipping), occ. layer of pink, medium to coarse crystalline with black inclusions, occ. dark green to black layer with dark green veins, white oxidation/encrustation on some partings, rust oxidation on vertical partings, high strength, slightly weathered to unweathered, very close to close spaced flat to dipping partings, occ. vertical partings, rough planar, tight to oxidized, variably very poor to excellent quality.

GRANITIC GNEISS: Light to medium grey, fine to medium crystalline, slight to heavy banding (dipping), high strength, slightly weathered to unweathered, with thin layers of schist at 1.7 m and 2.0 m, low strength, very close to close becoming moderate spaced flat to dipping partings, rough planar to smooth planar, tight to oxidized, good to excellent quality.

MIGMATITE: White and black, fine to medium crystalline, some distorted banding, with biotite concentrations, medium to high strength, unweathered, close spaced flat to dipping partings, smooth planar (mica schistosity), tight, excellent quality.

GRANITIC GNEISS: Light grey and pink, fine to medium crystalline, slight banding, occ. biotite concentrations, some coarse feldspar (pegmatite), high strength, unweathered, moderate to wide spaced flat partings, rough planar, tight to oxidized, excellent quality.

Originated: JFW  
Compiled: NR  
Checked: CN

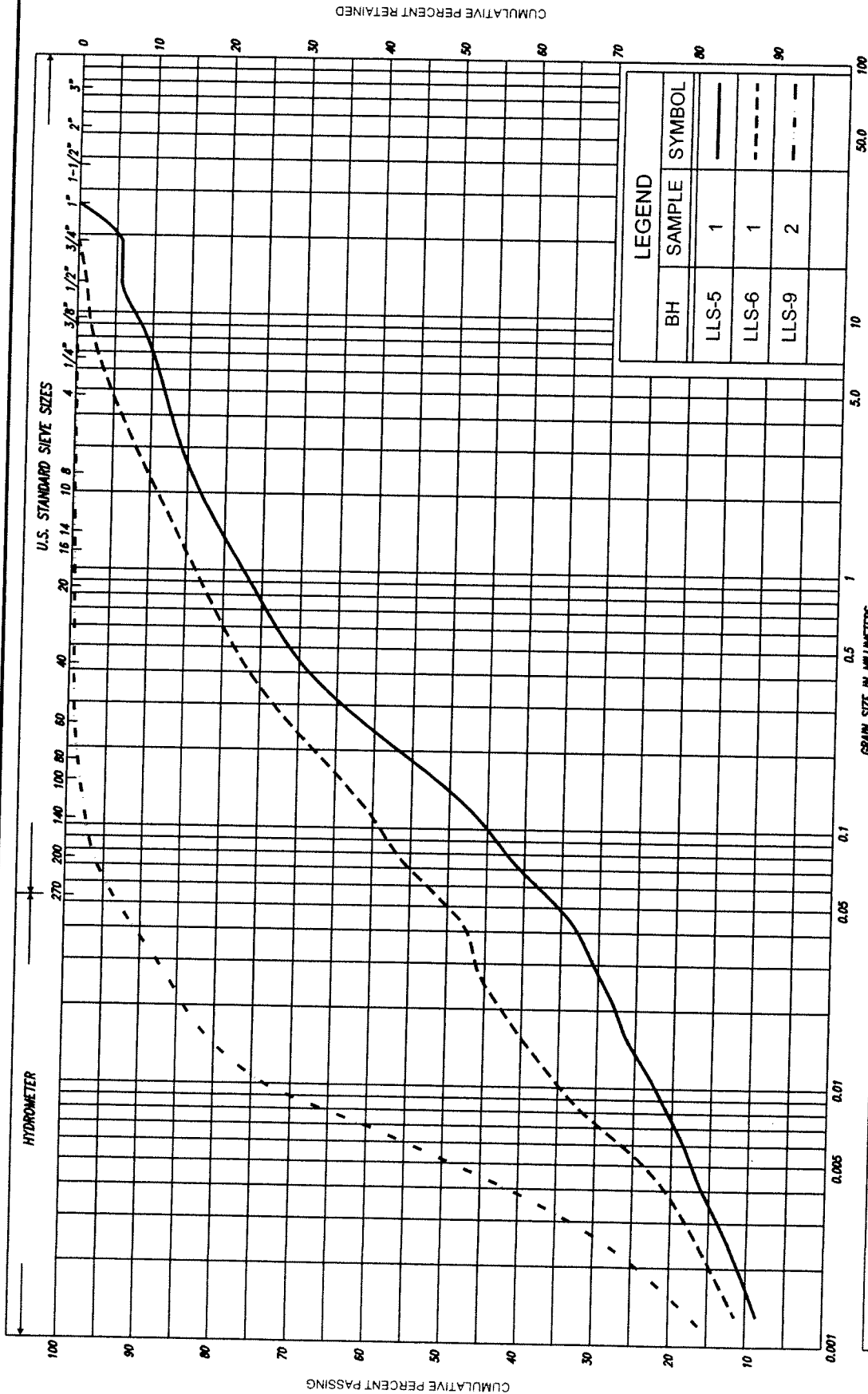


**TABLE A**  
**ROCK CORE DESCRIPTION**

CORE RECOVERY						CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION	
LLS-8	3	1.6 – 2.9	100	89	1.6 - 4.8	GRANITIC GNEISS: Light grey and pink, fine crystalline, slight banding, occ. biotite concentrations, some coarse feldspar, high strength, unweathered, close to moderate spaced flat to dipping partings, rough to smooth planar, tight to oxidized, good to excellent quality.	
	4	2.9 – 4.4	100	100			
	5	4.4 – 4.8	100	100			

NOTES: RQD: Rock Quality Designation

Originated: JFW  
 Compiled: NR  
 Checked: CN



# GRAIN SIZE DISTRIBUTION CLAYEY SILT, trace sand to sandy, trace to some gravel

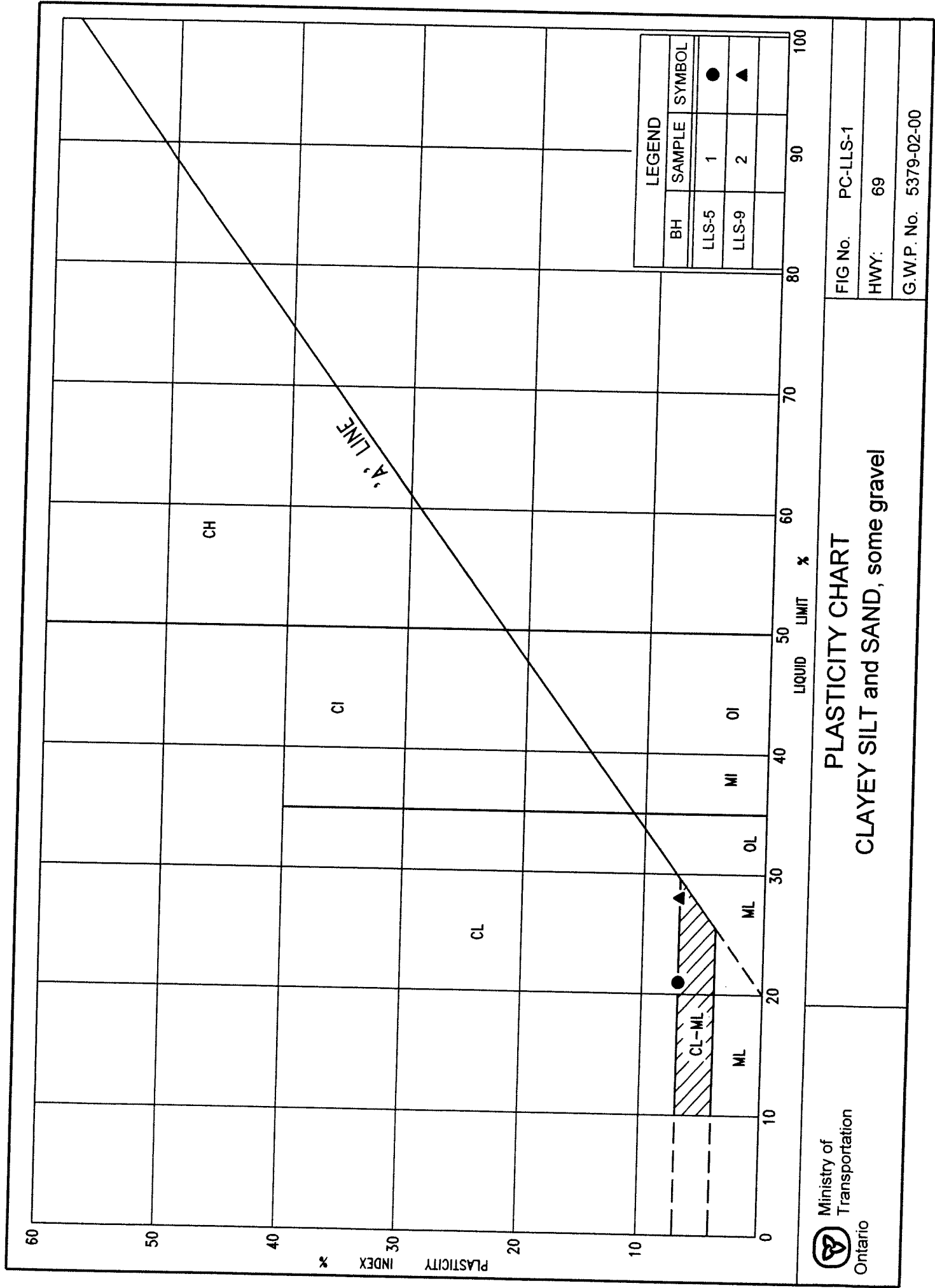
FIG No. GS-LLS-1

HWY: 69

G.W.P. No. 5379-02-00

U.S. BUREAU

GRAIN SIZE IN MILLIMETERS												5.0	10	50.0	100						
SILT & CLAY												FINE		MEDIUM		COARSE		GRAVEL		COB	
CLAY		FINE		MEDIUM		SILT		COARSE		FINE		MEDIUM		COARSE		GRAVEL		COB			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
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CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			
CLAY		FINE		MEDIUM		SILT		COARSE		V. FINE		FINE		MED.		COARSE		GRAVEL			



## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_u$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	F M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	$S_r$	%	DEGREE OF SATURATION	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$w_L$	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_p$	%	PLASTIC LIMIT	$D_n$	mm	n PERCENT - DIAMETER
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_s$	%	SHRINKAGE LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL	WTPL		WETTER THAN PLASTIC LIMIT	j	kN/m <sup>3</sup>	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No LLS-1

1 of 1

METRIC

G.W.P. 1071-10-01 LOCATION Hwy 69 (Hwy), Sta. 14+10 CL of 14.0m Ls. of CL req. ORIGINATED BY N.L.B.  
DIST 14 HWY no BOREHOLE TYPE Portable Power Auger COMPILED BY N.R.  
DATUM Sea Level DATE March 27, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100			
413.3	Ground Surface											
413.4	Topsoil											
413.4	Sandy silt, with gravel occ. cobbles and boulders Loose Brown/ Moist grey						223					
413.3	End of borehole Refusal on probable bedrock											
	* Borehole dry											



RECORD OF BOREHOLE No LLS-2

1 of 1

METRIC

G.W.P. 110-01-01 LOCATION Rwy 62 Newl, Sta. 14+10 over 18.2m Cr. of Cl. Med. ORIGINATED BY N.L.S.  
 DIST 14 HWY 29 BOREHOLE TYPE Manual Augering COMPILED BY N.R.  
 DATUM Geodetic DATE March 27, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
216.5	Ground Surface												
216.0	Topsoil												
215.5	End of borehole												
	Refusal on probable bedrock												
	* Borehole dry												

RECORD OF BOREHOLE No LLS-3

1 of 1

METRIC

G.W.P. 0605-01-01 LOCATION Hwy 401 New, Sta. 14+10 to 14+15, E. of 401 Exd. ORIGINATED BY N.S.B.  
 DIST 14 HWY 40 BOREHOLE TYPE Manual Augering COMPILED BY N.S.B.  
 DATUM Mean Sea Level DATE March 27, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>p</sub> — W — W <sub>L</sub>	WATER CONTENT (%) W	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES								
210.0	Ground Surface												
0.0	Topsoil												
210.0	End of borehole												
0.0	Refusal on probable bedrock												
	* Borehole dry												

RECORD OF BOREHOLE No LLS-4

1 of 1

METRIC

G.W.P. 11-01-06-01 LOCATION Hwy 10 Newell, Sta. 11-144 on E. side of H. Rd.  
DIST 34 HWY 69 BOREHOLE TYPE Portable Drill and Washboring ORIGINATED BY N.L.B.  
DATE 11/01/06 DATE April 17/07, 2007 COMPILED BY H.F.  
CHECKED BY C.W.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
108.3	Ground Surface													
108.0	Topsoil													
107.5	Boulders and cobbles													
107.0	Boulders and cobbles with gravel, with sand with silt		1	CS	1		204							
106.5							204							
106.3							267							
106.0	probable boulder or bedrock		2	RC NO	1		204							
105.2	End of Borehole													
4.1	Borehole terminated due to broken core bit on probable boulder or rock fracture. New borehole LLS-4A drilled 1.0m south.  Borehole advanced with portable equipment due to site access restrictions on steep slope.  * Borehole dry													

RECORD OF BOREHOLE No LLS-4A

1 of 1

METRIC

G.W.P. 040-10-01 LOCATION Hwy 49 (New), Sta. 14+143 to a 17.8m E. of P.C. Mag. ORIGINATED BY M.T.P.  
DIST 04 HWY 49 BOREHOLE TYPE Portable Drill and Washpiping COMPILED BY N.R.  
DATE Dec 2006 DATE April 1-6/14, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT MOISTURE CONTENT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
								20 40 60 80 100						GR SA SI CL	
209.3	Ground Surface														
0.3	Topsoil														
0.3	Boulders and cobbles														
	Boulders and cobbles with gravel, with sand with silt		1	CS	-										
207.2	Bedrock		2	CS	-										
2.1	Granitic Gneiss:		3	RC NQ	REC 100%									RQD 100%	
	light to medium grey and pink, occ. dark green to black		4	RC NQ	REC 95%									RQD 64%	
	slightly weathered to unweathered		5	RC NQ	REC 96%									RQD 0%	
	high strength		6	RC NQ	REC P1%									RQD 0%	
	variable very poor to excellent quality		7	RC NQ	REC 100%									RQD 17%	
			8	RC NQ	REC 100%									RQD 71%	
			9	RC NQ	REC 94%									RQD 13%	
202.9	End of Borehole														
5.4	Borehole advanced with portable equipment due to site access restrictions on steep slope.														
	* Borehole dry														

RECORD OF BOREHOLE No LLS-4B

1 of 1

METRIC

G.W.P. 6021-03-01 LOCATION Bayview Ave. 14th Ave. 10.0m E. of 71.0m ORIGINATED BY N.S.  
 DIST 14 HWY 69 BOREHOLE TYPE Manual Augering COMPILED BY N.S.  
 DATUM Geodetic DATE March 16, 2007 CHECKED BY C.M.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
100.0	Ground Surface													
0.0	Topsoil													
0.0	Cobbles and boulders													
0.0	End of borehole													
	Refusal on pickable boulders													
	Borehole dry													

RECORD OF BOREHOLE No LLS-4C

1 of 1

METRIC

G.W.P. 500-02-0 LOCATION Hwy 69 New, Sta. 14114 P.S. 11.00 E. of St. Catharines  
DIST 54 HWY 69 BOREHOLE TYPE Manual Augering ORIGINATED BY N.R.  
DATUM Tenders DATE March 15, 2007 COMPILED BY N.R.  
CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
103.4	Ground Surface													
7.4	Topsoil													
0.3	Cobbles and boulders													
	End of borehole													
	Refusal on probable boulders													
	* Borehole dry													

# RECORD OF BOREHOLE No LLS-5

1 of 1

**METRIC**

G.W.P. 61-07-02

LOCATION

May 6<sup>th</sup> Newl, Sta. 14-106 O/S 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57, 58, 59, 60, 61, 62, 63, 64, 65, 66, 67, 68, 69, 70, 71, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, 83, 84, 85, 86, 87, 88, 89, 90, 91, 92, 93, 94, 95, 96, 97, 98, 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120, 121, 122, 123, 124, 125, 126, 127, 128, 129, 130, 131, 132, 133, 134, 135, 136, 137, 138, 139, 140, 141, 142, 143, 144, 145, 146, 147, 148, 149, 150, 151, 152, 153, 154, 155, 156, 157, 158, 159, 160, 161, 162, 163, 164, 165, 166, 167, 168, 169, 170, 171, 172, 173, 174, 175, 176, 177, 178, 179, 180, 181, 182, 183, 184, 185, 186, 187, 188, 189, 190, 191, 192, 193, 194, 195, 196, 197, 198, 199, 200, 201, 202, 203, 204, 205, 206, 207, 208, 209, 210, 211, 212, 213, 214, 215, 216, 217, 218, 219, 220, 221, 222, 223, 224, 225, 226, 227, 228, 229, 230, 231, 232, 233, 234, 235, 236, 237, 238, 239, 240, 241, 242, 243, 244, 245, 246, 247, 248, 249, 250, 251, 252, 253, 254, 255, 256, 257, 258, 259, 260, 261, 262, 263, 264, 265, 266, 267, 268, 269, 270, 271, 272, 273, 274, 275, 276, 277, 278, 279, 280, 281, 282, 283, 284, 285, 286, 287, 288, 289, 290, 291, 292, 293, 294, 295, 296, 297, 298, 299, 300, 301, 302, 303, 304, 305, 306, 307, 308, 309, 310, 311, 312, 313, 314, 315, 316, 317, 318, 319, 320, 321, 322, 323, 324, 325, 326, 327, 328, 329, 330, 331, 332, 333, 334, 335, 336, 337, 338, 339, 340, 341, 342, 343, 344, 345, 346, 347, 348, 349, 350, 351, 352, 353, 354, 355, 356, 357, 358, 359, 360, 361, 362, 363, 364, 365, 366, 367, 368, 369, 370, 371, 372, 373, 374, 375, 376, 377, 378, 379, 380, 381, 382, 383, 384, 385, 386, 387, 388, 389, 390, 391, 392, 393, 394, 395, 396, 397, 398, 399, 400, 401, 402, 403, 404, 405, 406, 407, 408, 409, 410, 411, 412, 413, 414, 415, 416, 417, 418, 419, 420, 421, 422, 423, 424, 425, 426, 427, 428, 429, 430, 431, 432, 433, 434, 435, 436, 437, 438, 439, 440, 441, 442, 443, 444, 445, 446, 447, 448, 449, 450, 451, 452, 453, 454, 455, 456, 457, 458, 459, 460, 461, 462, 463, 464, 465, 466, 467, 468, 469, 470, 471, 472, 473, 474, 475, 476, 477, 478, 479, 480, 481, 482, 483, 484, 485, 486, 487, 488, 489, 490, 491, 492, 493, 494, 495, 496, 497, 498, 499, 500, 501, 502, 503, 504, 505, 506, 507, 508, 509, 510, 511, 512, 513, 514, 515, 516, 517, 518, 519, 520, 521, 522, 523, 524, 525, 526, 527, 528, 529, 530, 531, 532, 533, 534, 535, 536, 537, 538, 539, 540, 541, 542, 543, 544, 545, 546, 547, 548, 549, 550, 551, 552, 553, 554, 555, 556, 557, 558, 559, 560, 561, 562, 563, 564, 565, 566, 567, 568, 569, 570, 571, 572, 573, 574, 575, 576, 577, 578, 579, 580, 581, 582, 583, 584, 585, 586, 587, 588, 589, 590, 591, 592, 593, 594, 595, 596, 597, 598, 599, 600, 601, 602, 603, 604, 605, 606, 607, 608, 609, 610, 611, 612, 613, 614, 615, 616, 617, 618, 619, 620, 621, 622, 623, 624, 625, 626, 627, 628, 629, 630, 631, 632, 633, 634, 635, 636, 637, 638, 639, 640, 641, 642, 643, 644, 645, 646, 647, 648, 649, 650, 651, 652, 653, 654, 655, 656, 657, 658, 659, 660, 661, 662, 663, 664, 665, 666, 667, 668, 669, 670, 671, 672, 673, 674, 675, 676, 677, 678, 679, 680, 681, 682, 683, 684, 685, 686, 687, 688, 689, 690, 691, 692, 693, 694, 695, 696, 697, 698, 699, 700, 701, 702, 703, 704, 705, 706, 707, 708, 709, 710, 711, 712, 713, 714, 715, 716, 717, 718, 719, 720, 721, 722, 723, 724, 725, 726, 727, 728, 729, 730, 731, 732, 733, 734, 735, 736, 737, 738, 739, 740, 741, 742, 743, 744, 745, 746, 747, 748, 749, 750, 751, 752, 753, 754, 755, 756, 757, 758, 759, 760, 761, 762, 763, 764, 765, 766, 767, 768, 769, 770, 771, 772, 773, 774, 775, 776, 777, 778, 779, 780, 781, 782, 783, 784, 785, 786, 787, 788, 789, 790, 791, 792, 793, 794, 795, 796, 797, 798, 799, 800, 801, 802, 803, 804, 805, 806, 807, 808, 809, 810, 811, 812, 813, 814, 815, 816, 817, 818, 819, 820, 821, 822, 823, 824, 825, 826, 827, 828, 829, 830, 831, 832, 833, 834, 835, 836, 837, 838, 839, 840, 841, 842, 843, 844, 845, 846, 847, 848, 849, 8

ORIGINATED BY N.I.D.

DIST 54

HWY 13

BOREHOLE

BOREHOLE TYPE Continuous Flight Auger Borehole

COMPILED BY N.R.

DATUM 120251Z

DATE \_\_\_\_\_

March 27, 1937

CHECKED BY C.N.

[illegible]

RECORD OF BOREHOLE No LLS-6

1 of 1

METRIC

G.W.P. 024-05-01 LOCATION Hwy 401/Highway 401 at 25 km E. of CL W-9. ORIGINATED BY N.L.B.  
DIST 34 HWY 40 BOREHOLE TYPE C.F.S.S.A. and Rotary Drill Boring COMPILED BY N.L.B.  
DATUM Reduced DATE March 23, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
215.0	Ground Surface													
214.0	Topsoil													
213.0	Clayey silt sandy trace gravel, organics occ. cobbles and boulders													
204.0	Stiff Brown Wet		1	SS	10		204							3 39 41 10
203.0	Bedrock													
	Granitic Gneiss: slightly weathered to unweathered high strength		2	RC NQ	REC 100%		203							RQD 75%
	thin layers of schist at 1.7 and 2.0m depth (low strength)		3	RC NQ	REC 100%		202							RQD 100%
			4	RC NQ	REC 100%		201							RQD 93%
200.0	End of borehole													
4.0	Sample 1: Insufficient sample for Atterberg Limit test.  * Borehole dry upon completion of drilling  C.F.S.S.A. - Denotes Continuous Flight Solid Stem Augers													



RECORD OF BOREHOLE No LLS-7

1 of 1

METRIC

G.W.P. LLS-7 LOCATION Key - 4 New, Sta. 14+171.0 on Hwy. 10, W. of Q. Rd. ORIGINATED BY N.B.B.  
DIST 14 HWY 89 BOREHOLE TYPE C.F.C.P.A. and Rotary Core Drilling COMPILED BY N.B.B.  
DATUM Canadian DATE March 16, 2007 CHECKED BY J.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
204.7	Ground Surface							20 40 60 80 100							
0.0	Topsoil							20 40 60 80 100							GR SA SI CL
0.3	Clayey silt with sand, with gravel Very stiff Brown Moist		1	GS	16		204								
203.4	Sand partings														
0.3	Sandy silt, with gravel occ. cobbles and boulders		2	GS	50/100		203								
0.7	Compact Brown Wet Bedrock		3	RC NQ	REC 92%										RQD 92%
	Migmatite: white and black unweathered medium to high strength excellent quality		4	RC NQ	REC 100%		202								RQD 100%
	Granitic Gneiss: light grey to pink unweathered high strength excellent quality		5	RC NQ	REC 100%		201								RQD 100%
200.0	End of borehole						200								
4.0	Sample 2: sampler bouncing														
	* 2007 03 16														
	Water level measured after drilling														

## RECORD OF BOREHOLE No LLS-8

1 of 1

**METRIC**

G.W.P. 100-10-10

LOCATION

St. Louis, Mo. 11-17-08 11.5m Lt. of CI Vnd

ORIGINATED BY N.S.B.

DIST

100

HWY 100

BOREHOLE TYPE

S.F.A.C.A. and Rotary Corp. Dr: 2.00

COMPILED BY S.E.

DATUM

1997, 2000, 2002, 2003, 2004, 2005, 2006, 2007, 2008, 2009, 2010, 2011, 2012, 2013, 2014, 2015, 2016, 2017, 2018, 2019, 2020, 2021, 2022, 2023, 2024, 2025, 2026, 2027, 2028, 2029, 2030, 2031, 2032, 2033, 2034, 2035, 2036, 2037, 2038, 2039, 2040, 2041, 2042, 2043, 2044, 2045, 2046, 2047, 2048, 2049, 2050, 2051, 2052, 2053, 2054, 2055, 2056, 2057, 2058, 2059, 2060, 2061, 2062, 2063, 2064, 2065, 2066, 2067, 2068, 2069, 2070, 2071, 2072, 2073, 2074, 2075, 2076, 2077, 2078, 2079, 2080, 2081, 2082, 2083, 2084, 2085, 2086, 2087, 2088, 2089, 2090, 2091, 2092, 2093, 2094, 2095, 2096, 2097, 2098, 2099, 2100, 2101, 2102, 2103, 2104, 2105, 2106, 2107, 2108, 2109, 2110, 2111, 2112, 2113, 2114, 2115, 2116, 2117, 2118, 2119, 2120, 2121, 2122, 2123, 2124, 2125, 2126, 2127, 2128, 2129, 2130, 2131, 2132, 2133, 2134, 2135, 2136, 2137, 2138, 2139, 2140, 2141, 2142, 2143, 2144, 2145, 2146, 2147, 2148, 2149, 2150, 2151, 2152, 2153, 2154, 2155, 2156, 2157, 2158, 2159, 2160, 2161, 2162, 2163, 2164, 2165, 2166, 2167, 2168, 2169, 2170, 2171, 2172, 2173, 2174, 2175, 2176, 2177, 2178, 2179, 2180, 2181, 2182, 2183, 2184, 2185, 2186, 2187, 2188, 2189, 2190, 2191, 2192, 2193, 2194, 2195, 2196, 2197, 2198, 2199, 2200, 2201, 2202, 2203, 2204, 2205, 2206, 2207, 2208, 2209, 2210, 2211, 2212, 2213, 2214, 2215, 2216, 2217, 2218, 2219, 2220, 2221, 2222, 2223, 2224, 2225, 2226, 2227, 2228, 2229, 2230, 2231, 2232, 2233, 2234, 2235, 2236, 2237, 2238, 2239, 2240, 2241, 2242, 2243, 2244, 2245, 2246, 2247, 2248, 2249, 2250, 2251, 2252, 2253, 2254, 2255, 2256, 2257, 2258, 2259, 2260, 2261, 2262, 2263, 2264, 2265, 2266, 2267, 2268, 2269, 2270, 2271, 2272, 2273, 2274, 2275, 2276, 2277, 2278, 2279, 2280, 2281, 2282, 2283, 2284, 2285, 2286, 2287, 2288, 2289, 2290, 2291, 2292, 2293, 2294, 2295, 2296, 2297, 2298, 2299, 2300, 2301, 2302, 2303, 2304, 2305, 2306, 2307, 2308, 2309, 2310, 2311, 2312, 2313, 2314, 2315, 2316, 2317, 2318, 2319, 2320, 2321, 2322, 2323, 2324, 2325, 2326, 2327, 2328, 2329, 2330, 2331, 2332, 2333, 2334, 2335, 2336, 2337, 2338, 2339, 2340, 2341, 2342, 2343, 2344, 2345, 2346, 2347, 2348, 2349, 2350, 2351, 2352, 2353, 2354, 2355, 2356, 2357, 2358, 2359, 2360, 2361, 2362, 2363, 2364, 2365, 2366, 2367, 2368, 2369, 2370, 2371, 2372, 2373, 2374, 2375, 2376, 2377, 2378, 2379, 2380, 2381, 2382, 2383, 2384, 2385, 2386, 2387, 2388, 2389, 2390, 2391, 2392, 2393, 2394, 2395, 2396, 2397, 2398, 2399, 2400, 2401, 2402, 2403, 2404, 2405, 2406, 2407, 2408, 2409, 2410, 2411, 2412, 2413, 2414, 2415, 2416, 2417, 2418, 2419, 2420, 2421, 2422, 2423, 2424, 2425, 2426, 2427, 2428, 2429, 2430, 2431, 2432, 2433, 2434, 2435, 2436, 2437, 2438, 2439, 2440, 2441, 2442, 2443, 2444, 2445, 2446, 2447, 2448, 2449, 2450, 2451, 2452, 2453, 2454, 2455, 2456, 2457, 2458, 2459, 2460, 2461, 2462, 2463, 2464, 2465, 2466, 2467, 2468, 2469, 2470, 2471, 2472, 2473, 2474, 2475, 2476, 2477, 2478, 2479, 2480, 2481, 2482, 2483, 2484, 2485, 2486, 2487, 2488, 2489, 2490, 2491, 2492, 2493, 2494, 2495, 2496, 2497, 2498, 2499, 2500, 2501, 2502, 2503, 2504, 2505, 2506, 2507, 2508, 2509, 2510, 2511, 2512, 2513, 2514, 2515, 2516, 2517, 2518, 2519, 2520, 2521, 2522, 2523, 2524, 2525, 2526, 2527, 2528, 2529, 2530, 2531, 2532, 2533, 2534, 2535, 2536, 2537, 2538, 2539, 2540, 2541, 2542, 2543, 2544, 2545, 2546, 2547, 2548, 2549, 2550, 2551, 2552, 2553, 2554, 2555, 2556, 2557, 2558, 2559, 2560, 2561, 2562, 2563, 2564, 2565, 2566, 2567, 2568, 2569, 2570, 2571, 2572, 2573, 2574, 2575, 2576, 2577, 2578, 2579, 2580, 2581, 2582, 2583, 2584, 2585, 2586, 2587, 2588, 2589, 2590, 2591, 2592, 2593, 2594, 2595, 2596, 2597, 2598, 2599, 2600, 2601, 2602, 2603, 2604, 2605, 2606, 2607, 2608, 2609, 2610, 2611, 2612, 2613, 2614, 2615, 2616, 2617, 2618, 2619, 2620, 2621, 2622, 2623, 2624, 2625, 2626, 2627, 2628, 2629, 2630, 2631, 2632, 2633, 2634, 2635, 2636, 2637, 2638, 2639, 2640, 2641, 2642, 2643, 2644, 2645, 2646, 2647, 2648, 2649, 2650, 2651, 2652, 2653, 2654, 2655, 2656, 2657, 2658, 2659, 2660, 2661, 2662, 2663, 2664, 2665, 2666, 2667, 2668, 2669, 2670, 2671, 2672, 2673, 2674, 2675, 2676, 2677, 2678, 2679, 2680, 2681, 26

DATE \_\_\_\_\_

Mar 8, 2017

CHECKED BY C.N.

[illegible]

RECORD OF BOREHOLE No LLS-9

1 of 1

METRIC

G.W.P. 6061-10-1 LOCATION Bayview Ave., Sta. 14+191 o/s 3.5m W. of E. Ave. ORIGINATED BY N.R.  
DIST 14 HWY 74 BOREHOLE TYPE Continuous Flight Auger Solid Stem Augers COMPILED BY N.R.  
DATUM Geodetic DATE March 17, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
104.8	Ground Surface													
104.4	Sand and gravel with silt, occ. bobbles													
104.3	Grey Clayey silt trace sand, trace gravel		1	SS	12		204							
	Stiff Mottled Moist brown/grey		2	SS	11		203							
	occ. layers of silt													
200.5	End of borehole													
2.4	Refusal on probable bedrock													
	Borehole dry upon completion of drilling													
	Penetrometer test													

# METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES

CONT No

WP No 5261-05-01

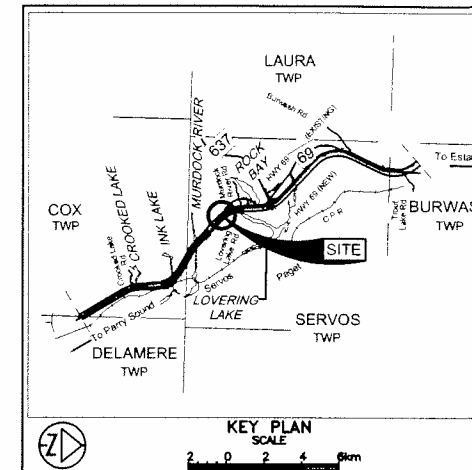
LOWERING LAKE ROAD OVERPASS SOUTHBOUND  
HIGHWAY 69

BOREHOLE LOCATIONS AND SOIL STRATA



SHEET

**PML Peto MacCallum Ltd.**  
CONSULTING ENGINEERS



**LEGEND**

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE Blows/0.3m (60° Cone, 475 J / blow)
- W L at time of investigation Mar-Apr 2007
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

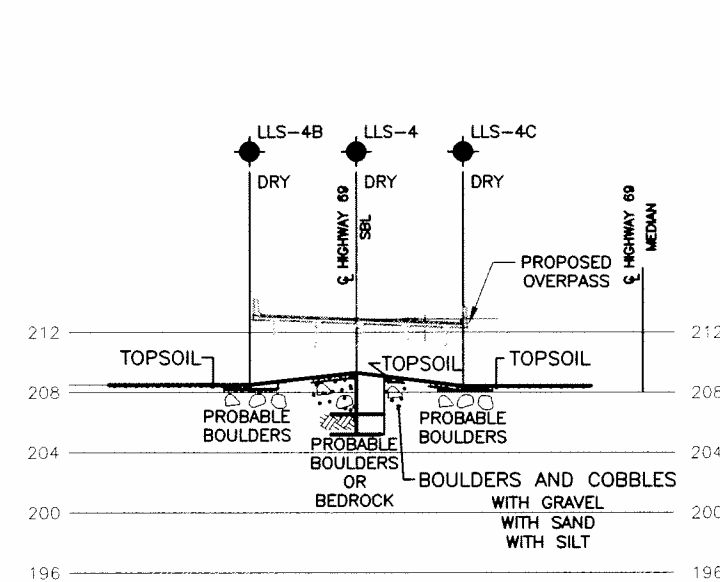
BH No	ELEVATION	STATION	OFFSET HWY 69 C. MED.
LLS-1	223.8	14+110	18.8m Lt.
LLS-2	216.5	14+130	18.8m Lt.
LLS-3	210.6	14+140	18.8m Lt.
LLS-4	209.3	14+144	18.8m Lt.
LLS-4A	209.3	14+143	18.8m Lt.
LLS-4B	208.5	14+144	25.8m Lt.
LLS-4C	208.4	14+144	11.8m Lt.
LLS-5	205.1	14+166	18.8m Lt.
LLS-6	205.1	14+171	25.8m Lt.
LLS-7	204.7	14+171	18.8m Lt.
LLS-8	204.4	14+171	11.8m Lt.
LLS-9	204.9	14+191	18.8m Lt.

**NOTE**  
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

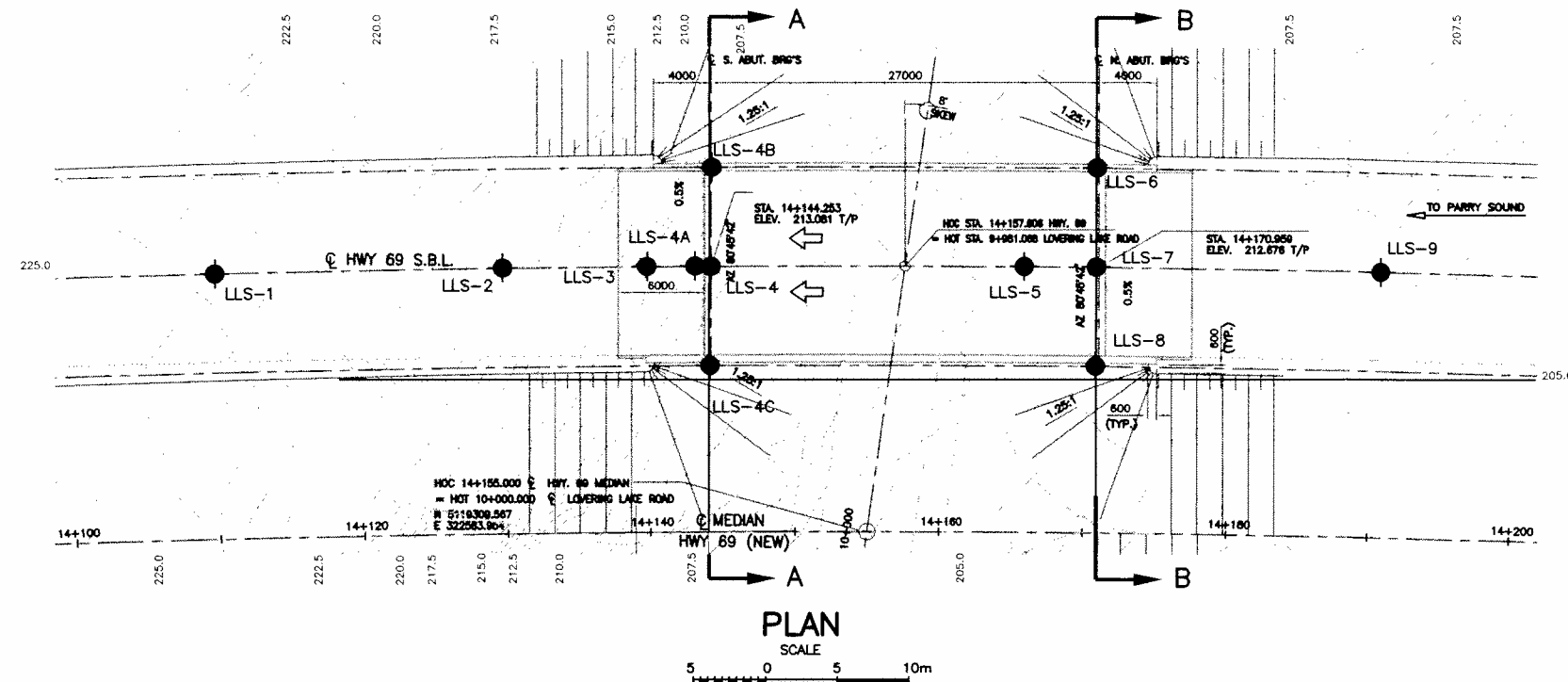
DATE	BY	DESCRIPTION

Geocres No. 411-213

HWY No	69	DIST	SUBURBY
SUBMD	MNR	CHECKED NR	DATE NOV. 13, 2007
DRAWN	NA	CHECKED CN	APPROVED BRG
			DWG

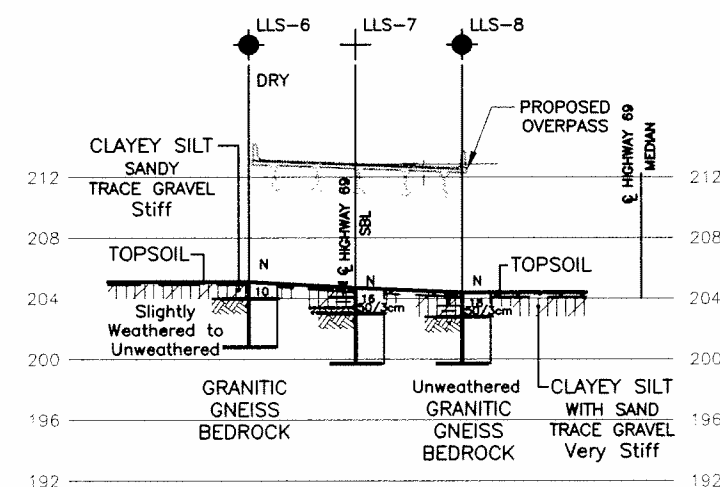


A-A



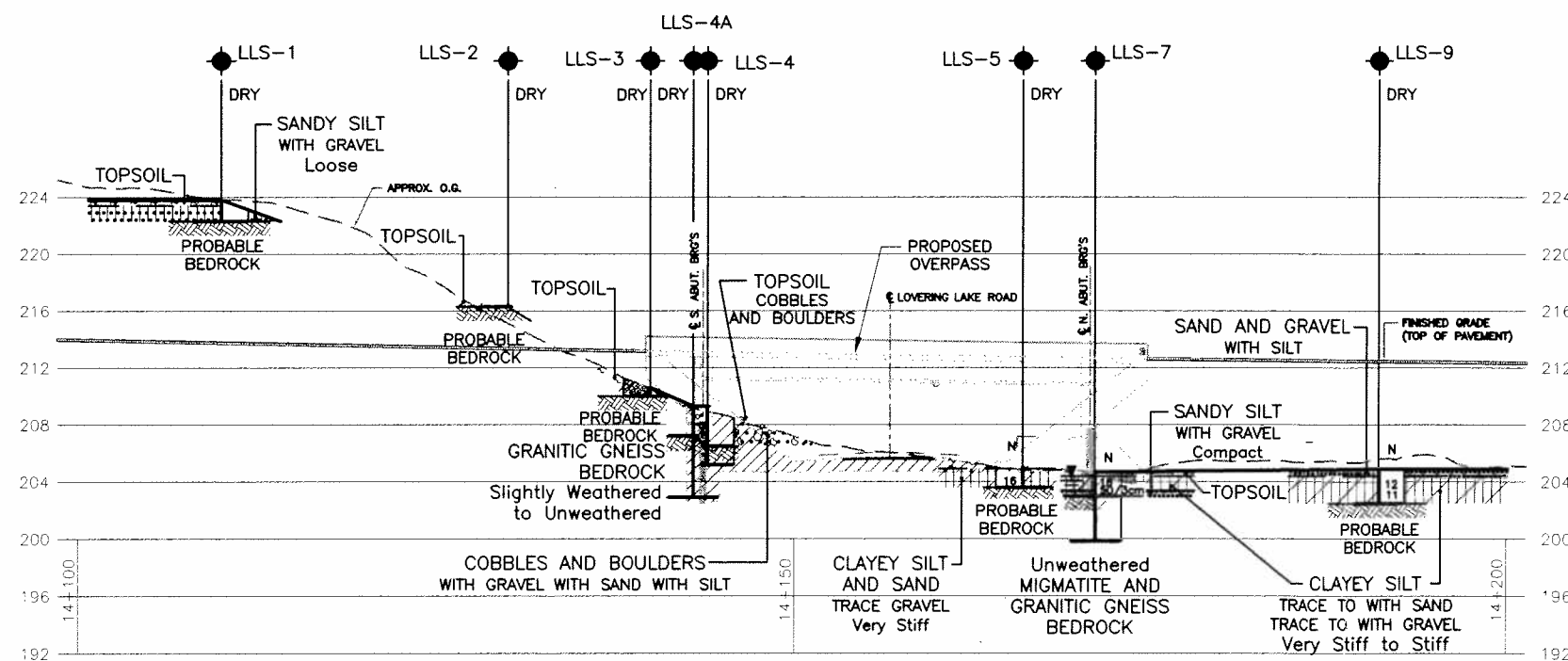
PLAN

SCALE  
0 5 10m



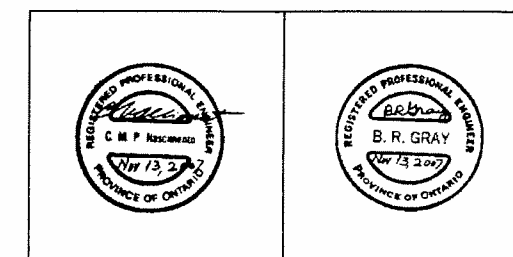
B-B

SCALE  
0 5 10m



PROFILE (HWY 69 SBL)

SCALE  
0 5 10m



REF No.: TSH DRAWINGS 42\_91088\_NBL\_SBL\_LOWERING\_GA.dwg  
470\_ph\_base\_nad83\_z12.dwg  
Phase1\_Contours.dwg

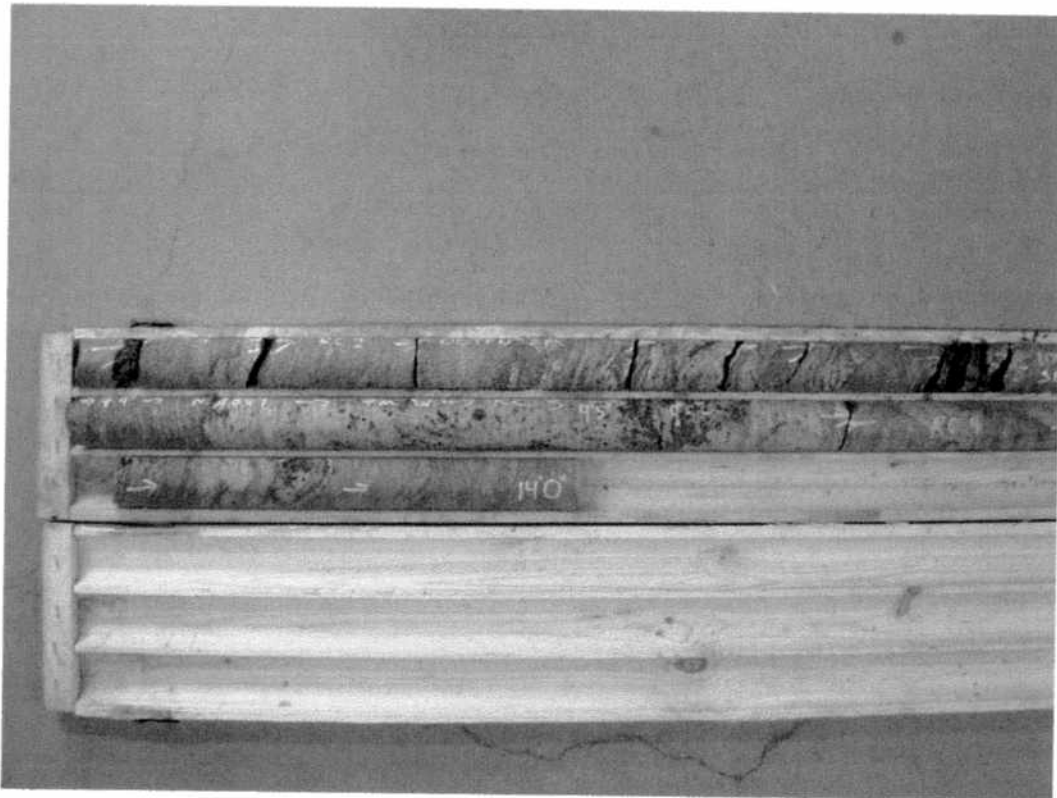
**NOTES:**

- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

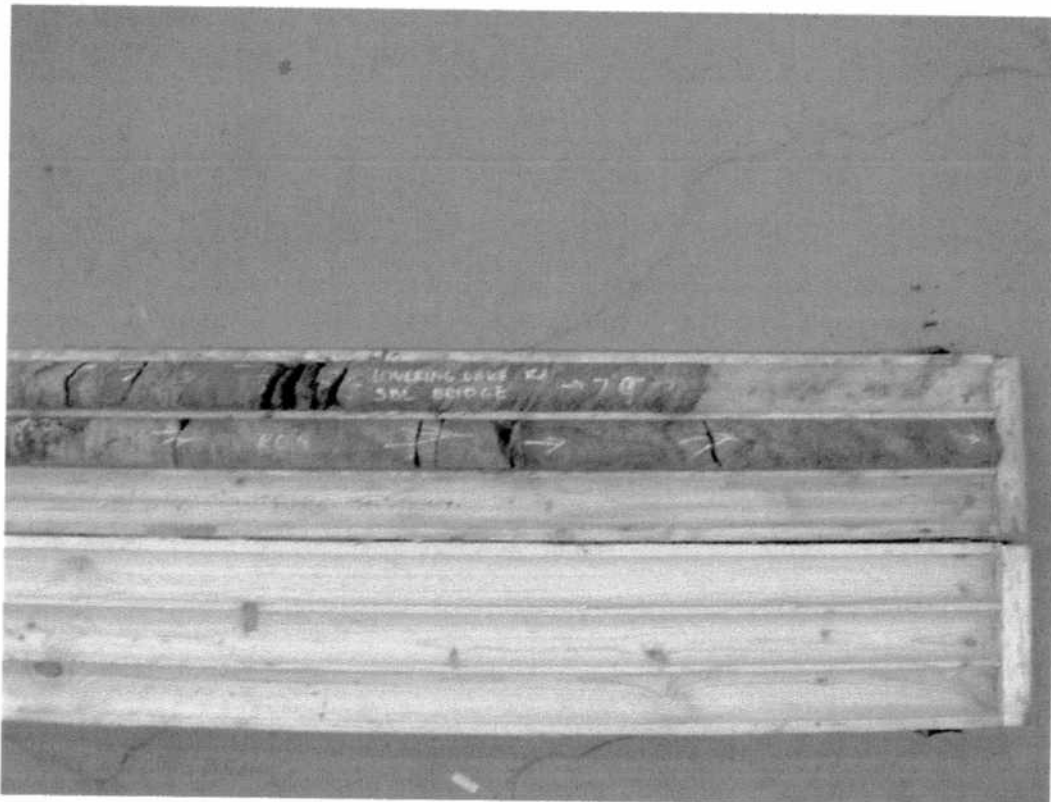


## **APPENDIX A**

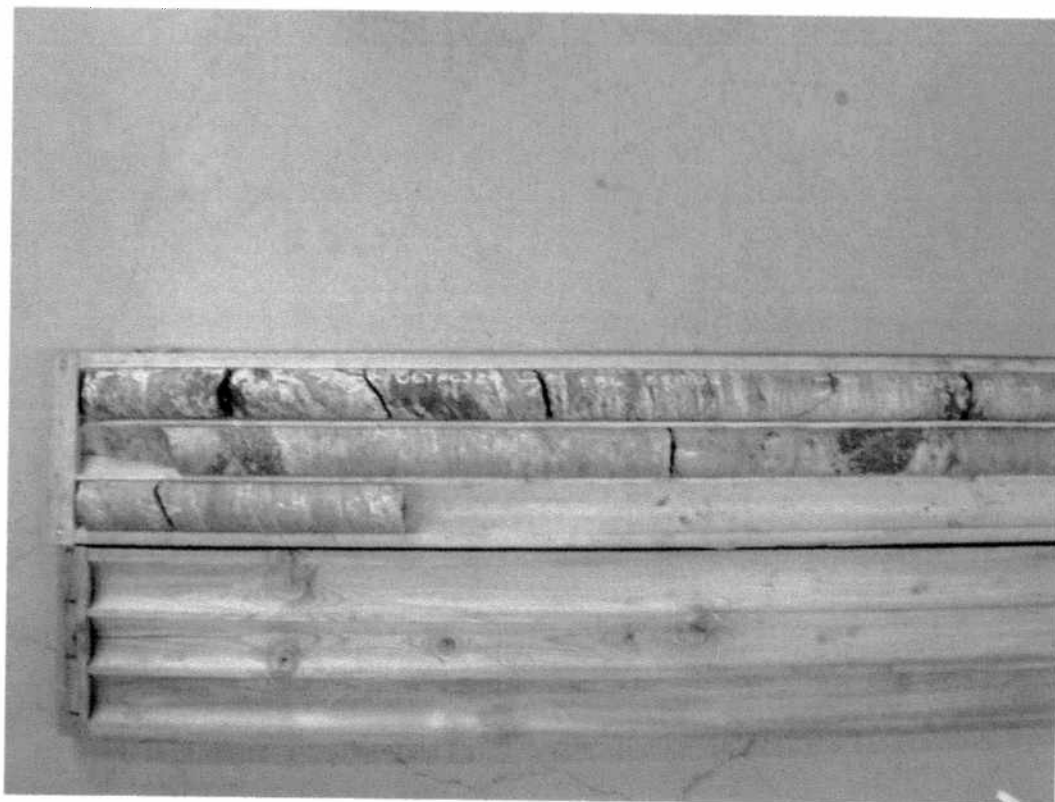
### Rock Core Photographs



Photograph 1: Rock core from borehole LLS-6



Photograph 2: Rock core from borehole LLS-6



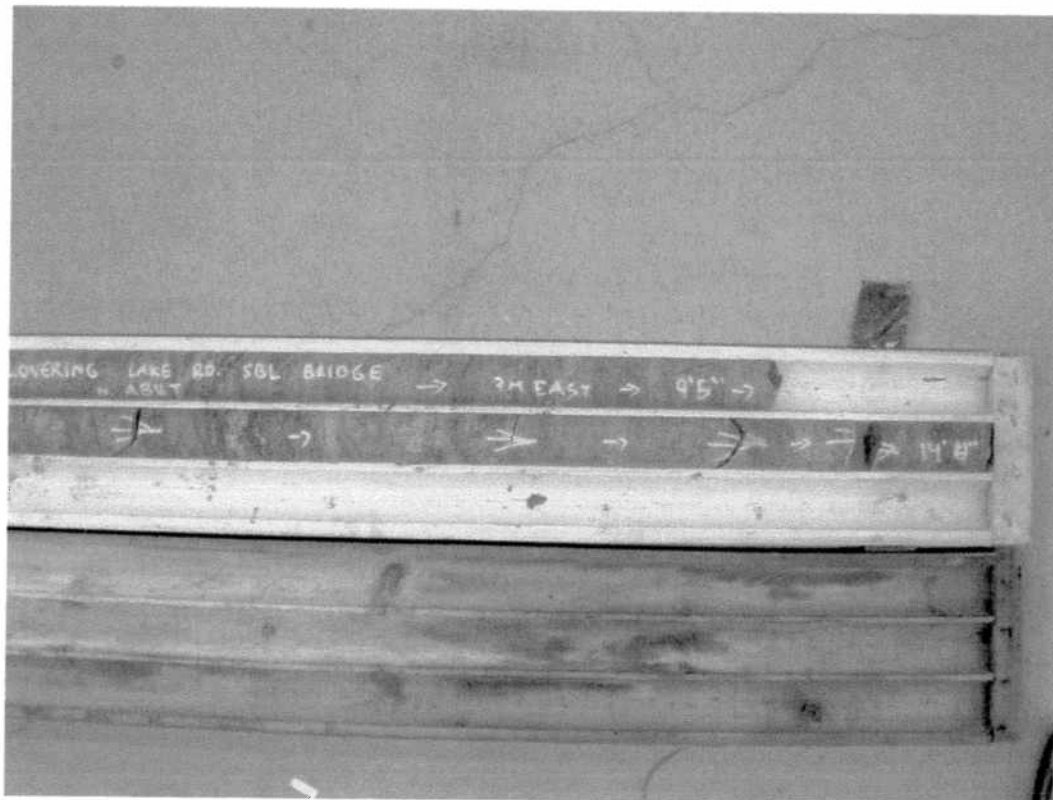
Photograph 3: Rock core from borehole LLS-7



Photograph 4: Rock core from borehole LLS-7



Photograph 5: Rock core from borehole LLS-8

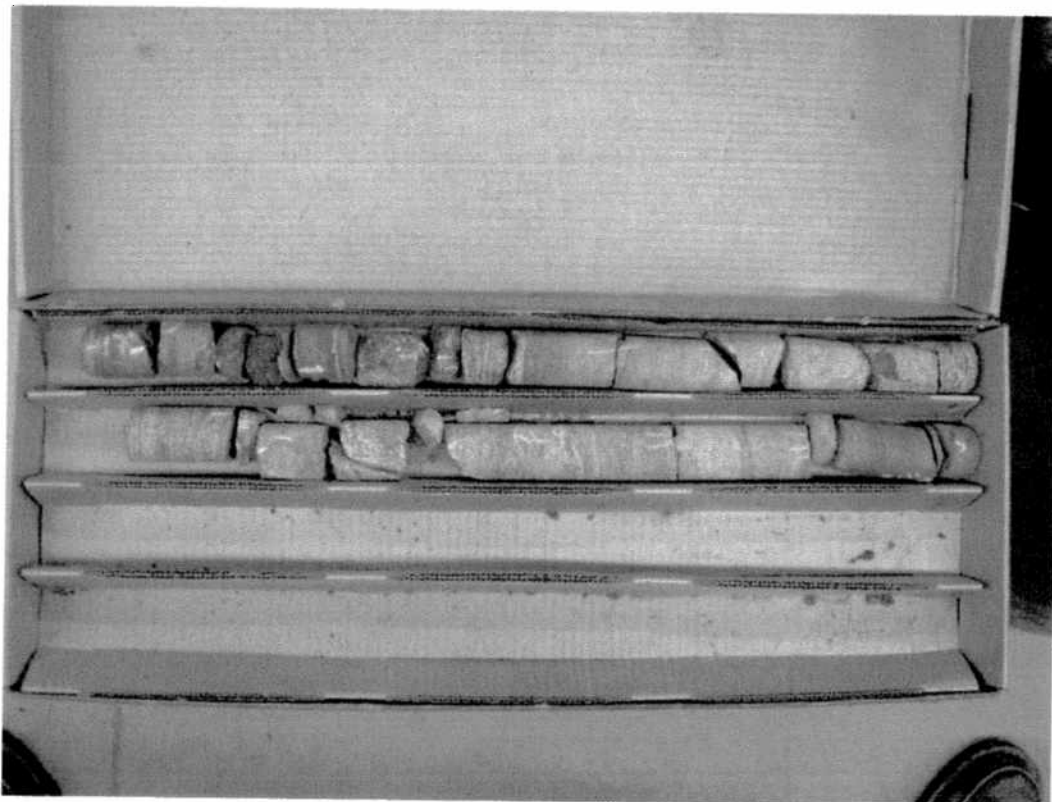


Photograph 6: Rock core from borehole LLS-8





Photograph 7: Rock core from borehole LLS-4A



Photograph 8: Rock core from borehole LLS-4A



## **APPENDIX B**

### Site Photographs



Photograph 1: Viewing west from the median of the proposed Highway 69 at Station 14+155. Note exposed bedrock and boulders in the photo (July 27, 2007).



Photograph 2: Viewing northeast from 7m west of centreline of the proposed SBL at approximately middle of the span length (July 27, 2007).



Photograph 3: Viewing southeast from 7m west of the centreline of the proposed SBL at approximately middle of the span length. Note boulders in the foreground of photo (July 27, 2007).



Photograph 4: Viewing towards south at south abutment of SBL. Note large boulders at the bottom half of the photo (July 27, 2007).



**FOUNDATION DESIGN REPORT**

**for**

**HIGHWAY 69**

**LOVERING LAKE ROAD OVERPASS SOUTHBOUND**

**SITE NO. 46-508/2, W.P. 5261-05-01**

**DISTRICT 54, SUDBURY**

***PHASE 1, STA. 12+200 TO 15+400***

***TOWNSHIP OF SERVOS***

PETO MacCALLUM LTD.  
165 CARTWRIGHT AVENUE  
TORONTO, ONTARIO  
M6A 1V5  
Phone: (416) 785-5110  
Fax: (416) 785-5120  
Email: toronto@petomaccallum.com

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Table 1 – List of Ontario Provincial Standard Documents Referenced in Report

Table 2 – Gradation Specification for Sand Fill in Pre-augered Holes at Integral Abutments

Figure 1 - Abutment on Compacted Fill Showing Granular 'A' Core

**FOUNDATION DESIGN REPORT**

for  
Lovering Lake Road Overpass Southbound  
Highway 69  
Site No. 46-508/2, WP 5261-05-01  
District 54, Sudbury, Ontario

*Phase 1, Sta. 12+200 to 15+400  
Township of Servos*

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**1. INTRODUCTION**

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations and the approach embankments for the proposed new Lovering Lake Road Overpass Southbound to be located about 44 km south of Sudbury, Ontario. The investigation was conducted for Totten Sims Hubicki Associates (TSH) on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed overpass southbound will carry the new Highway 69 southbound lanes (SBL) over the existing Lovering Lake Road between approximate Sta. 14+144.3 and 14+171.0.

The proposed overpass consists of a 27.0 m long, single span structure. The approach embankments to the north abutment will be raised 8.0 m above the existing grade to about elevation 212.7. The south approach will be constructed about 4.0 m above existing grade at elevation 213.1 and in front of a rock outcrop that will be cut down up to 11.0 m for the road platform.

The subsurface soil stratigraphy revealed in the boreholes generally comprised a surficial topsoil over a discontinuous layer of scattered cobbles and boulders (talus slope at the south abutment) and surficial topsoil or sand and gravel overlying cohesive stiff to very stiff clayey silt over a localized loose to compact sandy silt layer (north approach and abutment).

The soil cover mantled bedrock and bedrock outcrops were noted in particular a massive 20 m high rock outcrop immediately to the south of the south abutment. Numerous cobbles and boulders were encountered in the soil cover. The depth to the bedrock ranges from over 0.3 to 2.8 m below grade at the south abutment (elevation 207.2 and lower) which are about 5.9 m lower



than the proposed bridge deck. The bedrock depth ranges from 1.1 to 1.7 m at the north abutment (elevations 202.8 to 204.0) which are 8.7 to 9.9 m below the bridge deck. The quality of the typical granitic gneiss bedrock is generally good to excellent at the north abutment and variable very poor to excellent quality at the south abutment.

Based on the encountered subsurface conditions and the height of the embankments over the existing ground surface, the overpass foundations may be founded on spread footing placed on bedrock, native soil or engineered fill. Alternatively, deep foundations may be used provided that the rock is excavated where required at the abutments to allow for an adequate pile length.

The presence of numerous boulders within the native soils, in particular within the talus slope deposit found at the south abutment indicate that the installation of drilled case-in-place concrete caissons will not be practical. The boulders should also be considered when excavating the south abutment area for the installation of piles for the integral abutment solution. The trench through the talus slope deposit in the south abutment should be sloped at 1H:1V in view of potential voids and/or loose zones within the deposit. In bedrock, the walls may be cut nearly vertical.

The "red flag" issues outlined in the preceding paragraph and the recommended methods of overcoming these issues noted in the following sections of the report are intended to alert and aid the designer and the contractor. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the MTO for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable construction quality remain the responsibility of the contractor.

All elevations in this report are expressed in metres. A list of the Ontario Provincial Standard documents referenced in this report is enclosed in Table 1.





## **2. FOUNDATIONS**

### **2.1 General**

It is considered that the loose sand and gravel, and stiff clayey silt are compressible, therefore are not suitable for the bridge foundations. Also, the talus slope containing numerous boulders at the south abutment may contain loose zones and it is not considered an adequate founding subsoil.

It is considered feasible to support the foundations of the proposed abutment on spread footings constructed on bedrock at the south abutment and on the very stiff clayey silt or underlying bedrock at the north abutment. The bridge may also be founded on spread footings constructed on engineered fill. The use of caissons to support foundations on rock is not practical at this site due to the presence of numerous boulders in the soil cover that would cause installation difficulties.

The bedrock surface at the boreholes drilled for the abutments of the proposed Highway 69 SBL overpass was found at or within 5.9 and 9.9 m of proposed grade at the bridge deck.

Conventional, semi-integral and integral abutments are considered feasible at this site with some rock excavation at the south/north abutment to obtain adequate for pile height, based on the foregoing considerations. The type of foundation employed to support the foundation loads of the proposed structure and the system of bridge design will be dictated by structural considerations, economic considerations and construction constraints. From a foundation engineering perspective, use of integral abutments supported on piles driven to bedrock is the preferred type of the abutment foundations.

All footings and/or pile caps subject to frost action should be provided with 2.0 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 0.6 m of soil cover. Footings bearing directly on bedrock do not require protection from frost.



The seismic site coefficient for the conditions at this site is 1.0 (soil profile Type 1, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6).

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The liquefaction potential of the silts and sands at the site was assessed using the procedure suggested by Seed and Idriss (1971) and, on this basis it is considered that liquefaction of the granular soils is unlikely (clause 4.6.2 of the CHBDC).

## 2.2 Footings on Native Soils or Bedrock

The reference founding levels for spread footings placed on the bedrock at the south abutment and on the native soils or bedrock at the north abutment are provided on the following table:

FOUNDATION UNIT	SUBGRADE TYPE	REFERENCE LEVELS	
		ELEVATIONS	DEPTHS* (m)
South abutment	Bedrock	207.2**	2.1 to 2.8**
North abutment	Very stiff Clayey Silt	204.1 to 204.9	0.2 to 0.3
	Compact Sandy Silt	203.4	1.3
	Bedrock	202.8 to 204.0	1.1 to 1.7

\* Depth from existing ground surface. The minimum 2.0 m frost protection was not considered.

\*\* Sloping rock at the south abutment and the proposed road cut will require that the footing be placed at a lower elevation, estimated elevation 205.0.

Since the thickness of the native soils under the proposed north abutment is relatively small, these materials may be stripped to place the footing on the underlying bedrock surface.



The following geotechnical resistances should be used for the design of the spread footings:

FOUNDATION UNIT	SUBGRADE TYPE	FACTORED BEARING RESISTANCES AT ULS (kPa)	GEOTECHNICAL RESISTANCES AT SLS (kPa)
South abutment	Bedrock	8,000	Not applicable
North Abutment	Very stiff Clayey Silt	300	175
	Compact Sandy Silt	650	275
	Bedrock	10,000*	Not applicable

The factored bearing resistance at ULS at the south abutment was reduced by 20% due to the presence of very poor bedrock within the rock mass.

Considering the bedrock to be non-yielding, the design will not be governed by settlement criteria since the loading required to produce 25 mm deformation is much larger than the factored resistance at ULS. The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

A minimum footing width of 2.0 m and groundwater at about elevation 204.1 were considered for the ULS computation.

Construction of the footings should be performed and monitored in accordance with SP 902S01 to verify the competency of the founding surface.

Mass concrete could be placed on the bedrock surface to provide a level founding surface for the footings and/or be employed to raise the subgrade to the design founding level of the footings on bedrock. The need to expand the plan area at the base of the mass concrete to provide for stress distribution (2V:1H), place reinforcing steel in the mass concrete and/or use high strength concrete to prevent overstressing of the mass concrete will be dictated by the actual thickness of the mass concrete and structural design considerations.



Subject to these comments, the bearing resistance provided for footings bearing on bedrock is considered to be appropriate for mass concrete with an unconfined compressive strength of at least 35 MPa. If the actual bearing pressure is less than 10000 kPa, the compressive strength of the concrete could be reduced in direct linear proportion to the actual bearing pressure (minimum value of 2500 kPa).

Comments concerning excavation of the bedrock, if required to found the footings at a level lower than previously indicated, are provided in subsequent sections of the report.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the founding soils or bedrock. The following parameters should be used for sliding resistance of cast-in-place concrete spread footings on native soils.

PARAMETER	COMPACT SANDY SILT	VERY STIFF CLAYEY SILT
Friction angle, degrees	32	0
Cohesion, kPa	0	150
Unit weight, kN/m <sup>3</sup>	20.5	20.0

An unfactored friction factor of 0.7 is considered to be suitable at this site on the "rough bedrock surfaces" (asperity height of at least 25 mm). The factored horizontal resistance at ULS of the bedrock is considered to be 5000 kPa.

The lateral resistance of footings founded on bedrock could be increased, if required, by installing shear keys, sockets or anchors into the bedrock (SP 999S26). The increased lateral resistance will be provided by the shear strength of the steel dowels, the horizontal resistance of the bedrock and the horizontal component of tensile forces developed in any inclined anchors. A greater frictional resistance between the footing and rock may be achieved if the anchors are prestressed to increase the vertical pressure.



If dowels are employed, a NSSP should be included in the tender documents to provide specific direction for the contractor during installation and testing of the dowels. Fractured rock should be removed from these areas. A NSSP should also be prepared for the shear keys.

Design, installation and testing of the anchors should be conducted in accordance with SP 999S26 and clause 6.10.4 (CHBDC). If anchors are installed, a factored bond stress at the rock/grout interface of 1.4 MPa at ULS (a resistance factor of 0.4 is applied for a minimum 35 MPa grout) is recommended for design. The total capacity of a group of closely spaced anchors may be less than the summed capacities of the individual anchors; the impact of anchor interaction should be assessed if the spacing is less than one-fifth of the anchor length.

### **2.3 Footings on Structural Fill**

Construction of the abutment footings on structural fill placed in the approach embankment could also be employed to support the foundation loads. The structural fill should comprise Ontario Provincial Standards Specifications (OPSS) Granular A material placed in maximum 200 mm thick layers, compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density.

The existing native soil at the south abutment which comprises of a bouldery talus slope that may contain voids or loose zones, and the stiff compressible clayey soils locally encountered at the north abutment (borehole LLS-6) should be removed. The structural fill should extend to the bedrock surface or be placed on the compact sandy silt or on the very stiff clayey silt encountered at the north abutment. The geometry should also extend out to a plane inclined downwards at 45° to the horizontal originating at least 1 m away from the top of the footing. This scheme is illustrated in the appended Figure 1. The extent of the fill should be established by a site specific survey prior to placement of the fill.

Footings should not be constructed on rock fill. However, rock fill may be placed adjacent to the Granular 'A' core shown in Figure 1.



The recommended bearing resistance for a 2.0 m wide footing constructed on structural fill is as follows:

Factored Bearing Resistance at ULS	900 kPa
Bearing Resistance at SLS	350 kPa

The engineered fill should be at least 2.0 m thick at the north abutment. At the south abutment, the bearing resistance is independent of fill thickness because the engineered fill should be placed directly on the bedrock.

The resistance at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 2.0 m and a groundwater level at elevation 203.5 was assumed for computation of the ULS resistance.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the structural fill. An unfactored friction factor of 0.7 is recommended for footings on the granular structural fill.

## **2.4 Pile Foundation**

It is considered feasible to have the north abutment founded on piles driven through a granular fill pad placed on the existing native soils. To found the south abutment on piles a trench should be excavated/blasted through the talus slope material and into the underlying bedrock where the pile length will be less than 5 m. This soil and bedrock removal will be required to allow design and construction of an integral abutment design for the overpass.

The general pile foundation design recommendations are provided on the following paragraphs followed by additional recommendations for integral abutment foundations.



The estimated range of reference founding levels for piles for the north abutment and the maximum levels of the rock excavation at the south abutment where piles should be driven to refusal on bedrock are provided on the following table:

LOCATION	DEPTH TO ROCK (m) (*)(**)	PILE FOUNDING ELEVATION (**)	RELEVANT BOREHOLES
North Abutment	1.1 to 1.7	202.8 to 204.0	LLS-6, LLS-7 and LLS-8
South Abutment	2.1	207.2	LLS-4A

(\*) A +1.0 m variation of the average depth to rock should be allowed for construction estimation purposes.

(\*\*) Rock excavation to lower levels will be required to establish the piles at an elevation providing at least 5 m of free pile length for integral abutments. Sloping bedrock at the south abutment will require excavation to provide an adequately level pile bearing surface for a conventional piled abutment foundations.

The recommended factored axial resistance at ultimate limit states (ULS) for the pile sections listed below is considered to be appropriate.

**FACTORED AXIAL RESISTANCE AT ULS, kN**

HP 310 x 110	2000
HP 310 x 152	2800
HP 360 x 108	2000
HP 360 x 152	2800

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the bedrock to be non-yielding and the relatively short pile length required, the design is not expected to be governed by settlement since the required loads causing appreciable deformation of the pile and/or bedrock are much larger than the ULS factored capacity.



It is considered that negligible down drag force will develop on the piles due to negative skin friction since the consolidation of the shallow native clayey silt/silty clay and overlying cohesionless soils under the embankment loading will occur during construction.

Refer to Section 4 for a discussion and recommendations on the treatment of approach embankment settlements.

The presence of numerous cobbles/boulders was identified above bedrock at depth in boreholes LLS-4, LLS-4A, LLS-4B and LLS-4C. Since the risk of damage during driving is considered to be high the boulders should be removed from the intended pile foundation area before driving the piles. The excavation should be backfilled with granular materials with a maximum nominal size of 75 mm compacted to 95% of the standard Proctor maximum dry density. Nevertheless, a NSSP should be prepared to advise the contractor of the presence of boulders at this site. The NSSP is required to ensure that more comprehensive engineering supervision is required than is called for in SP 903S01.

The compacted granular fill pad placed as a working platform for construction equipment during installation of the abutment piles should comprise OPSS Granular A material to allow installation of the piles without damage. Alternative granular materials could be employed provided the maximum particle size does not exceed 75 mm.

The piles will be driven through 4 to 6 m of the compacted granular fill pad and the underlying native soils that typically comprise compact silty sand and stiff to very stiff cohesive soils. It is considered, based on PML extensive experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed. Since the piles will be driven to bedrock, a specific set is not provided.

The piles will set on or into bedrock and should be equipped with "Rock Points" in accordance with SP 903S01. The Titus H Bearing Pile Points, Rock Injector Model (Titus Points) should be used at the north abutment since the slope of the bedrock revealed at the north borehole locations





ranges from 2 to 20°. For the purpose of uniformity, the piles for the south abutment, which will be installed in a rock trench, should also use the same type of rock points.

## **2.5 Integral Abutments on Piles**

The bedrock surface level at the south abutment is at elevation 207.2 that is 5.9 m below the proposed top of pavement elevation 213.1.

The depth/level of excavation of a trench into rock to accommodate the use of integral abutments will be dictated by structural design details. The excavation width should be at least 1 m wider than the plan area of the piles; side slopes in the soil cover and in the rock should be excavated as indicated in section 5 of this report. The excavation should be backfilled with Granular A, following the procedures outlined in the section titled "Approach Embankments". Further comments concerning bedrock excavation are provided in the section titled "Excavation and Groundwater Control".

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.



## 2.6 Lateral Resistance

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile below the annular space referred to previously. The assessed horizontal passive resistance values for the pile sections noted previously is as follows:

	<b>STIFF/ VERY STIFF CLAYEY SILT</b>		<b>COMPACT SANDY SILT/SAND</b>		<b>GRANULAR BACKFILL</b>	
Pile Section	HP310	HP360	HP310	HP360	HP310	HP360
Factored Lateral Resistance at ULS, kN	170	200	115	160	120	170
Lateral Resistance at SLS, kN	90	110	45	60	50	70

The assessed values of lateral resistance for a lateral movement of 10 mm assume that the piles are driven through the native undisturbed soils or through compacted granular materials placed as recommended previously. If greater resistance is required, batter piles should be installed.

To evaluate the point of contraflexure, the coefficient of horizontal subgrade reaction,  $k_s$  (MN/m<sup>3</sup>) should be computed using the following equations:

a) Cohesionless Soils (Terzaghi, 1955)

$$k_s = n_h z/b$$

where  $n_h$  = coefficient related to soil density  
 = 10.0 MN/m<sup>3</sup> for granular backfill  
 = 2.0 MN/m<sup>3</sup> for native silty/sandy soils above groundwater table (elevation 204.1)  
 = 1.3 MN/m<sup>3</sup> for native silty/sandy soils below elevation 204.1  
 $z$  = depth, m  
 $b$  = pile width, m

b) Cohesive Soils (Davison, 1970)

$$k_s = 67 \tau_u/d$$

where  $\tau_u$  = Undrained shear strength of clayey silt, use 150 kPa  
 $d$  = Pile diameter or width, m



The cohesionless soil parameter is applicable to all granular fill and to the native silty sand layers at the north abutment. The cohesive soil parameters apply to the cohesive silty clay and clayey silt units encountered at the north abutment.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters/widths. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor,  $R$ , as follows:

PILE SPACING IN DIRECTION OF LOADING $d$ = PILE DIAMETER OR WIDTH	SUBGRADE REACTION REDUCTION FACTOR, $R$
8d	1.00
6d	0.70
4d	0.40
3d	0.25

## 2.7 Comparison of Foundation Alternatives

Caisson foundations were not considered to be feasible due to the presence of numerous boulders above the bedrock. A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives discussed in the preceding paragraphs is presented below.

### Footings on Native Soil or Bedrock

#### Advantages

- Ease of installation
- Lower cost than deep foundations
- May be used with semi-integral abutments

#### Disadvantages

- Requires removal of all boulders from foundation footprint (south abutment)
- Possible need to remove fractured rock
- Possible need for mass concrete



### **Footings on Engineered Fill**

#### **Advantages**

- Ease of installation
- Lower cost than deep foundations
- Reduced height of abutment
- Fractured rock need not be excavated
- May be used with semi-integral abutments

#### **Disadvantages**

- Construction of engineered fill pad requires wider area than footings on bedrock
- Requires construction of an engineered fill pad

### **Piles to Bedrock**

#### **Advantages**

- High bearing resistance
- Used for integral abutments

#### **Disadvantages**

- Requires construction of a fill pad ahead of the approach embankment construction
- Higher installation cost than spread footings
- Requires excavation of trench in rock where free pile length is less than 5.0 m

### **3. ABUTMENT WALLS**

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure,  $p$  (kPa), may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation:

$$p = K (\gamma h + q) + C_p + C_s$$

where  $K$  = coefficient of lateral earth pressure (dimensionless)

$\gamma$  = unit weight of free-draining granular material,  $\text{kN/m}^3$

$h$  = depth below final grade, m

$q$  = surcharge load, kPa, if present

$C_p$  = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)

$C_s$  = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)

where  $\phi$  = angle of internal friction of retained soil ( $35^\circ$  for Granular B Type II)

$\delta$  = angle of friction between the soil and wall ( $23.5^\circ$  for Granular B Type II)



The seismic site coefficient for the conditions at this site was provided in Section 2.1.

Hydrostatic pressures were not included in the equation since free-draining OPSS granular material or rockfill will be used as backfill behind the wall according to OPSD-3121.150. The following parameters are recommended for design:

PARAMETER	GRANULAR A, GRANULAR B TYPE II or TYPE III	ROCKFILL
Angle of Internal Friction, degrees	35	42
Unit weight, kN/m <sup>3</sup>	22.8	18.0
Coefficient of Active Earth Pressure, $K_a$	0.27	0.20
Coefficient of Earth Pressure At Rest, $K_o$	0.43	0.33
Coefficient Passive Earth Pressure, $K_p$	3.69	5.04

Refer to MTO Report SO-96-01 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments. The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures. The earth pressure coefficients should be reviewed if the slope of the backfill exceeds  $10^\circ$  to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load ( $q$  in the preceding equation).

The magnitude of the passive resistance and active pressure is dependent on the actual lateral movement of the structure toward and away from the adjacent soil, respectively. We refer to Figure C6.16 (Clause C6.9.1) of the CHBDC for these computations. The backfill should be considered as medium dense sand for this project.

A weeping tile system (SP 405F03, OPSD-3102.100 and OPSD-3190.100) and/or weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet.



Backfilling adjacent to retaining structures should be carried out in conformance with OPSD 3101.150 and 3101.200 for granular or rock backfill at abutments.

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to SP 105S10 for additional information in this regard.

#### **4. APPROACH EMBANKMENTS**

The existing topsoil and compressible clayey soils within the backfill zone of the abutments and retaining walls should be excavated prior to placement of the backfill.

The level of the approach embankments will be raised about 3.8 m (south) and 8.0 m (north) above the existing ground surface. It is anticipated that the new embankments will be constructed with rock fill. The field investigation indicates that the existing native soils generally comprise heterogeneous mixture of cohesionless sand and gravel, sandy silt, and cohesive clayey silt and variable amounts of gravel, cobbles and boulders in a compact/stiff to very stiff condition. Surficial topsoil was encountered in most of the boreholes. The existing soils under the approach embankment are relatively thin (0.3 to 2.8 m thick) and cover shallow bedrock. Construction of the embankment fill remote from the abutments on the existing soils considered to be feasible.

The embankments should be constructed in accordance with OPSD 201.020, 202.010, 208.010 and SP 206S03. The side slopes of approach embankments should be inclined no steeper than 2 horizontal to 1 vertical (2H:1V) for earth fill and 1.25H:1V for rockfill. Where the height of the embankment is greater than 8 or 10 m for earth or rockfill, respectively, a 2 m wide mid-height berm will be required.

Where slope flattening is proposed, a drainage gap should be provided in accordance with OPSD 202.020. Where slopes are flattened to eliminate the need for a guide rail, a granular infilled drainage gap should be provided. OPSS Granular B Type II or Type III should be used for the drainage gap.



Since the new embankments will be constructed on a 0.3 to 2.8 m thick soil layer overlying bedrock the platform width should be widened by a minimum of 2 m each side in accordance with the Northeastern Region Engineering Directive (NRE 98-200).

It is considered that the approach embankments constructed in accordance with these recommendations will be stable. Settlement of the embankment fill due to consolidation of the underlying bedrock will be negligible.

Settlement of the road surface during and following completion of construction will result from two mechanisms – consolidation of the existing native soils below the embankment fill and “self weight” consolidation of the embankment fill.

The settlement of new rock fill should be in the order of 25 to 40 mm if placed in accordance with the requirements of SP 902S01 and OPSS 501 (Method A).

Consolidation of the underlying soil is expected to be in the order of 5 to 10 mm. Hence, the total consolidation settlement could be 30 to 50 mm. The settlement is expected to be essentially complete within three to six months following placement of the new fill.

Earth fill slopes, if utilized, should be protected against surface erosion by sodding (OPSS 571) and suitable vegetation. Refer to OPSS 572 for time constraints and type of seed and mulch required.

## **5. EXCAVATION AND GROUNDWATER CONTROL**

Excavation for construction of footings founded on bedrock will extend through about 2.1 m and 1.1 to 1.7 m thick in-situ soil cover at the south and north abutments, respectively. Cobbles and boulders should be expected in the excavations.

The talus slope containing boulders at the south abutment and the compact sandy silt and stiff clayey silt soils materials are classified as Type III soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria although the very stiff clayey silt deposits are



considered Type II soils, the higher soil type number should be considered for planning of the excavation slopes. Therefore, temporary cut slopes above the groundwater level inclined at 45° to the horizontal should be used. Flatter side slopes (about 3H:1V) will be required below the water level in cohesionless soils.

A large excavator equipped with a tiger-toothed bucket in conjunction with a jackhammer or hoe ram is the preferred method of excavation to shallow depths in rock (scaling). Conventional rock excavation techniques such as blasting (OPSS 120) may also be required. The actual equipment required and method of excavation within the bedrock will be dependent upon the geometry of the cut and relative depth of excavation into the bedrock. Mass concrete could be employed to level minor variations in the bedrock surface as mentioned previously.

It is important that blasting of the rock is controlled (SP 299F06) to prevent fracturing and/or disturbance of the bedrock surface on which footings will be founded. Any overblasting/overexcavation should be made the sole responsibility of the contractor and all loosened rock resulting from blasting operations is to be removed by mechanical means.

Near vertical sidewalls may be utilized in excavations in bedrock. Examination of the sidewalls and removal of any loosened rock fragments should be carried out continually for the safety of workers.

Groundwater at the south abutment was not encountered. Groundwater at the north abutment was found at 0.6 and 0.9 m below ground surface, elevation 203.5 and 204.1. Subject to the groundwater level at the time of construction, it is considered feasible to employ sump pumps to control groundwater seepage into the excavations for construction of the south abutment footings and the retaining wall foundations.

Surface water run-off should be diverted away from excavation to ensure that the foundations are constructed in the dry.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.





6. CLOSURE

The report was prepared by Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer. It was reviewed by Mr. B.R. Gray, MEng., P.Eng., MTO Designated Principal Contact carried out an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

A handwritten signature in cursive script, reading "C. M. P. Nascimento", is positioned above the printed name.

Carlos M. P. Nascimento, P. Eng.  
Senior Project Engineer



A handwritten signature in cursive script, reading "Brian R. Gray", is positioned above the printed name.

Brian R. Gray, MEng., P.Eng.  
MTO Designated Principal Contact



CN/BRG:nr-lnr



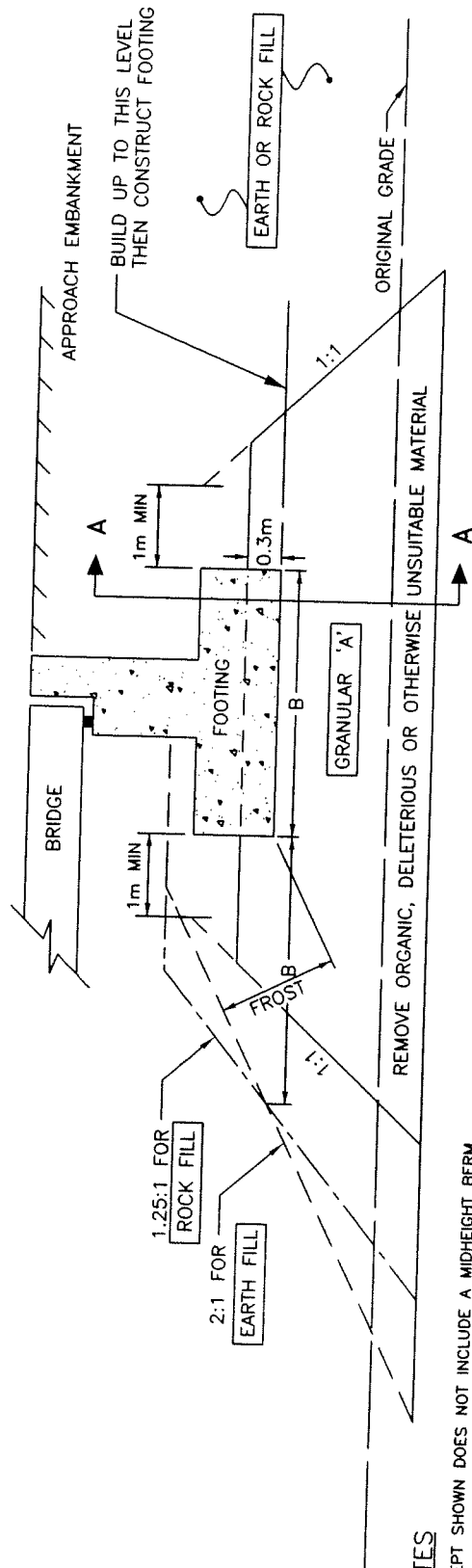
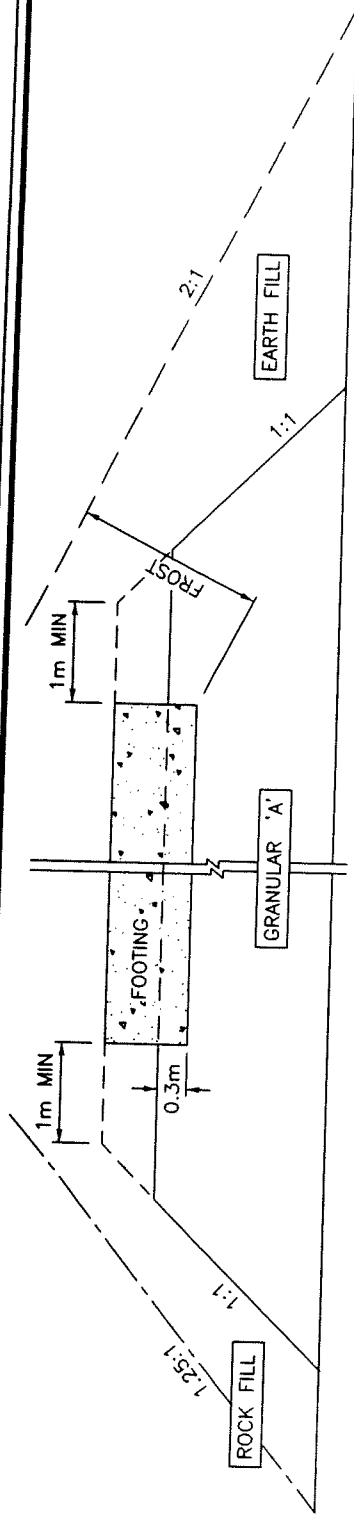
**TABLE 1**  
**LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT**

DOCUMENT	TITLE	DATE
OPSS 120	General Specification for the Use of Explosives	November 2003
OPSS 501	Construction Specification for Compacting	November 2005
OPSS 571	Construction Specification for Sodding	November 2001
OPSS 572	Construction Specification for Seed and Cover	November 2003
SP 105S10	Construction Specification for Compaction	November 2004
SP 206S03	Construction Specification for Grading	November 2006
SP 299F06	Rock Excavation (Controlled Blasting)	December 2001
SP 405F03	Construction Specification for Pipe Subdrains	November 2006
SP 902S01	Excavation and Backfilling of Structures	June 2006
SP 903S01	Construction Specification for Piling	November 2006
SP 999S26	Requirements for Design, Installation and Testing of Temporary and Permanent Pre-Stressed Anchors in Soil and Rock	November 2006
OPSD-201.020	Rock Grading-Divided Rural	November 2005
OPSD-202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment	November 2005
OPSD-202.020	Drainage Gap for Slope Flattening on Rock or Granular Embankment	November 2005
OPSD-208.010	Benching of Earth Slopes	November 2003
OPSD-3101.150	Minimum Granular Backfill Requirements - Abutments	November 2005
OPSD-3102.100	Walls – Abutment Backfill Drain	November 2005
OPSD-3101.200	Rock Backfill Requirements - Abutments	November 2005
OPSD-3121.150	Walls – Retaining, Backfill Minimum Granular Requirement	November 2005
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail	November 2005
NRE 98-200	Northeastern Region Directive - Platform Widening	October 28, 1998
NSSP	Dowels Into Concrete	December 2002
NSSP	Shear Keys	N/A
NSSP	Presence of Boulders	N/A



**TABLE 2**  
**GRADATION SPECIFICATION FOR SAND FILL IN**  
**PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS**

MTO SIEVE DESIGNATION		PERCENTAGE PASSING BY MASS
2 mm	#10	100
600 µm	#30	80 – 100
425 µm	#40	40 – 80
250 µm	#60	5 – 25
150 µm	#100	0 – 6



#### NOTES

1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

**FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE**